Geotechnical Report for
Energinet Eltransmission A/S

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EXECUTIVE SUMMARY

This report presents the geotechnical results for the geotechnical site investigation undertaken within the Bornholm I and Bornholm II Offshore Wind Farm site, Denmark.

Energinet Eltransmisson A/S commissioned Gardline Limited (Gardline) to support the development of a new offshore wind farm through the acquisition of geotechnical data which will inform the basis for evaluation of methods for wind turbine foundation design and installation. The objective will be met through acquiring ground information through both Seabed CPTU data and Down the Hole (DtH) Sampling and CPTU. PS logging was also conducted at selected locations.

The site investigation was located in the Baltic Sea, partially within Danish Territorial waters and partially on the Danish Continental Shelf. The site is approximately 20km south (Bornholm I) and south-west (Bornholm II) of the Island of Bornholm, covering a total area of around 652km^2. Figure I shows the project location boundaries. Figure II and Figure III show an overview of the Bornholm I and Bornholm II fields and investigation points. Water depths were measured to be 28m to 47m at Bornholm I and 32m to 56m at Bornholm II.

Figure I – Bornholm I and Bornholm II OWF Survey Area

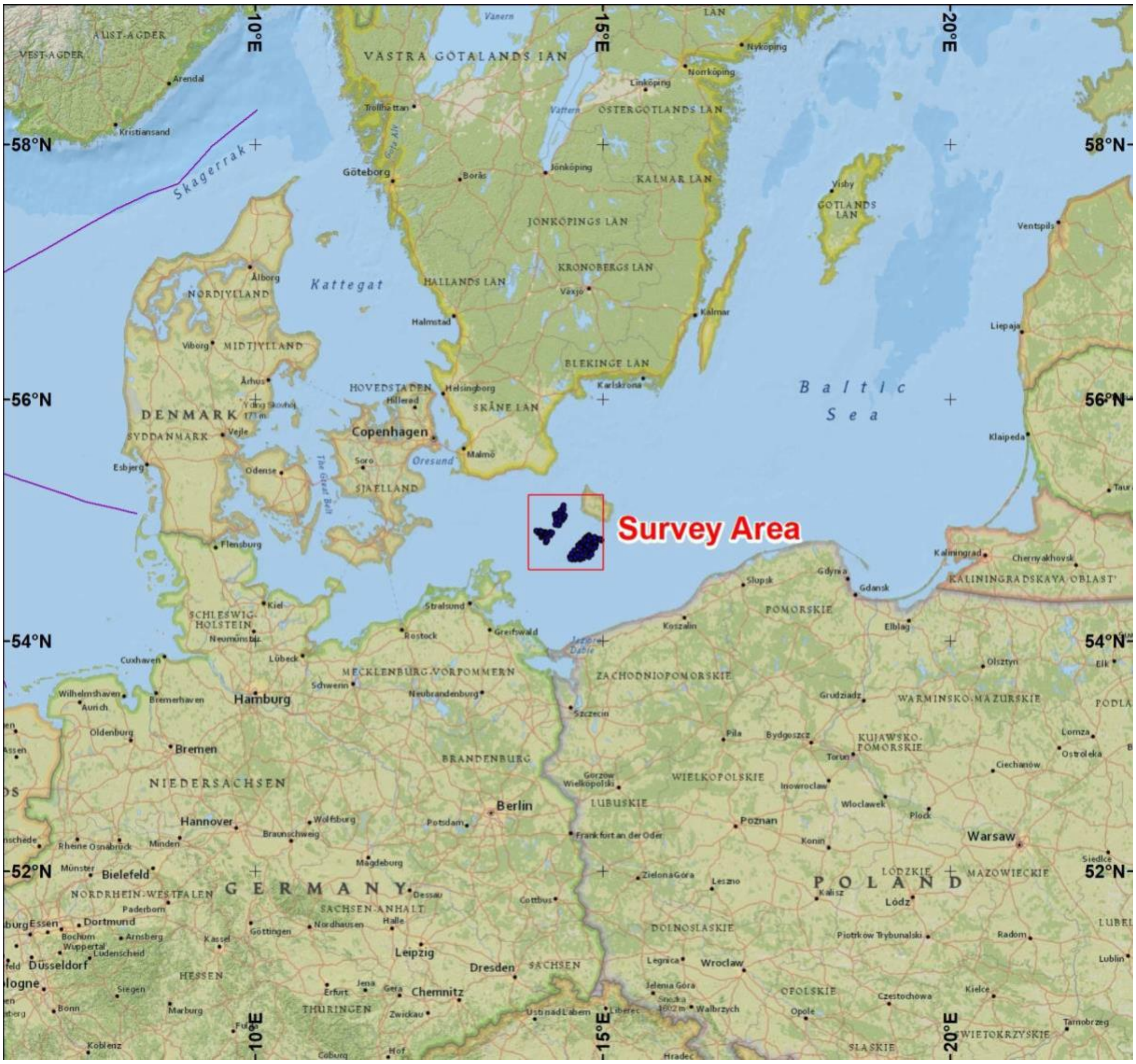


Figure II – Bornholm I OWF Overview

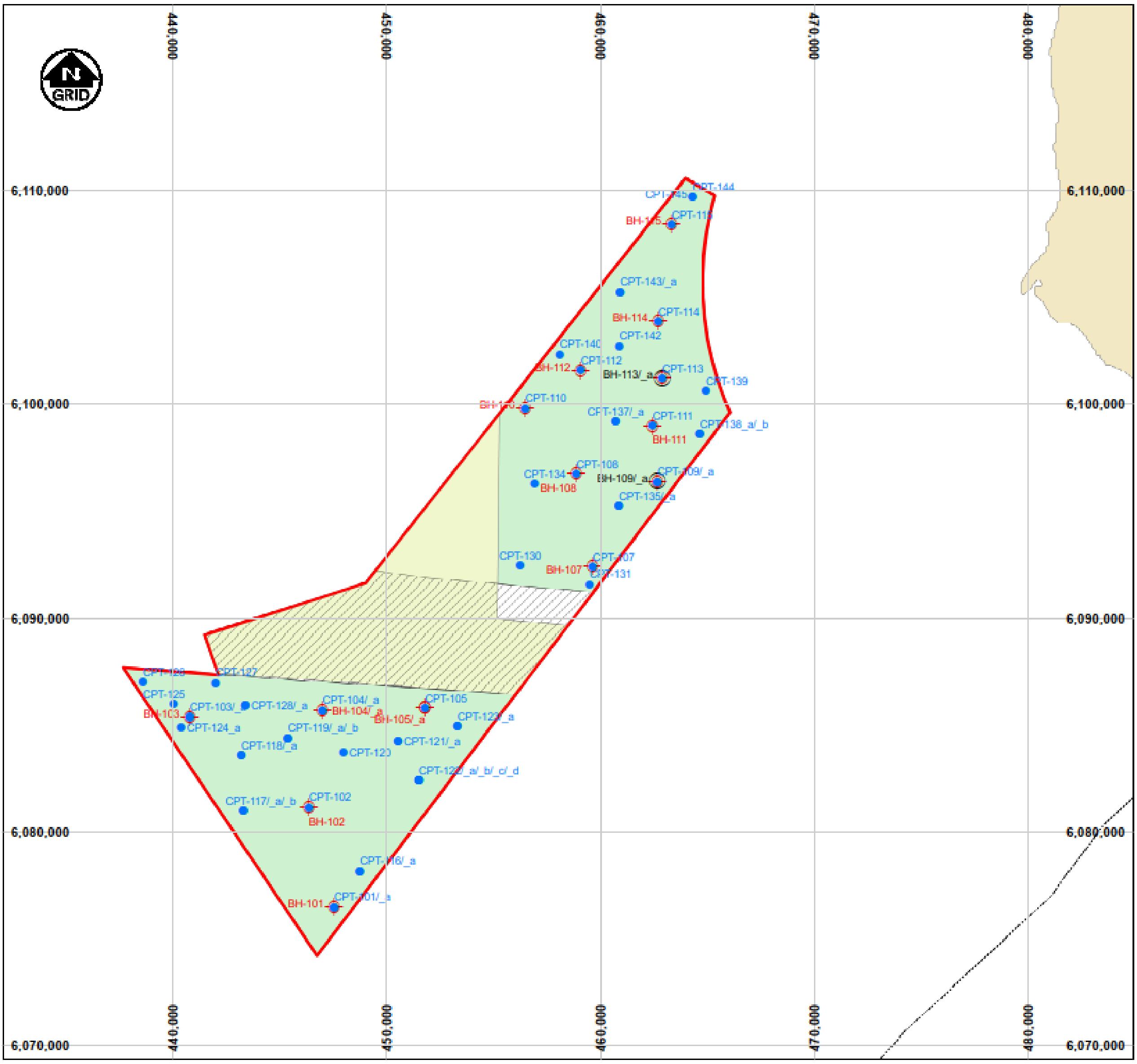
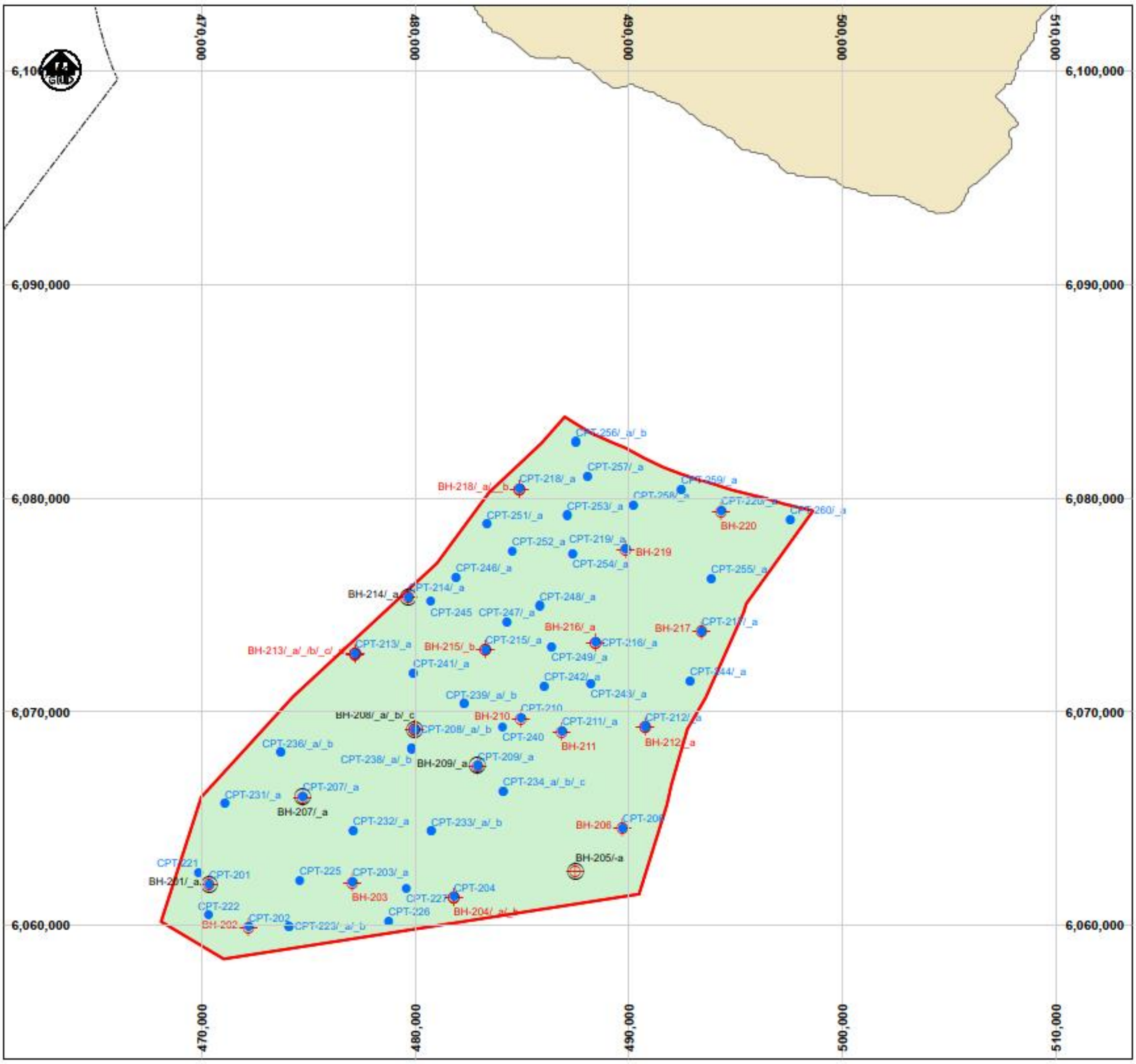


Figure III – Bornholm II OWF Overview



Individual location maps are presented in APPENDIX 1, showing each borehole and seabed CPT test relative to each location. Where borehole sampling locations are shown on the maps, these represent locations where only samples were acquired and no downhole CPT testing was conducted.

Survey operations were conducted with respect to ETRS89 Datum, UTM Grid Zone 33N. The grid system is presented in Eastings and Northings (m).

All water depths are reduced to Mean Sea Level (MSL) in metres and can be found in APPENDIX 1 together with location coordinates.

The objectives of the geotechnical investigation were to acquire geotechnical *in situ* data, samples and information as basis for evaluation of preliminary wind turbine foundation design and installation, as well as preliminary design of wind turbine configuration. This will be supported through the completion of an onshore laboratory testing programme and the results reported to inform the feasibility of achieving the safe installation and operation of the Bornholm I and Bornholm II OWF.

Two geotechnical drilling vessels were mobilised to conduct the scope of the project fieldwork, the M.V. Kommandor Susan and M.V. Ocean Vantage. Table I and Table II provide a summary of the fieldwork, including the number of locations completed, fieldwork dates and vessel details. All discussions regarding field operations, including encountered issues are discussed in the Field Operations report.

Table I – Fieldwork Summary – M.V. Kommandor Susan

Fieldwork Summary	
Survey Vessel	M.V. Kommandor Susan
Mobilisation Port	Gdansk, Poland
Mobilisation Date	30/03/2022
Fieldwork Dates	03/04/2022 – 06/10/2022
Sampling, Coring and CPTU Composite Boreholes	28 (+ 2 Bump overs)
Seabed CPTU 200kN Locations	24 (+ 14 Bump overs)
Seabed CPTU Neptune 5000 Locations	64 (+ 46 Bump overs)
PS Logging Locations	3
Demobilisation Port	Køge, Denmark
Demobilisation Date	06/10/2022

Table II – Fieldwork Summary – M.V. Ocean Vantage

Fieldwork Summary	
Survey Vessel	M.V. Ocean Vantage
Mobilisation Port	Gdansk, Poland
Mobilisation Date	01/09/2022
Fieldwork Dates	02/09/2022 – 26/09/2022
Sampling, Coring and CPTU Composite Boreholes	5 (+ 1 Bump Over and + 2 Re-drills*)
PS Logging Locations	3
Demobilisation Port	Gdansk, Poland
Demobilisation Date	27/09/2022
*Locations where Geobore-S coring was performed to compare sample quality on already acquired PQ3 locations.	

Soil and rock samples acquired from the field were subject to an onshore laboratory testing programme including classification, consolidation, chemical and strength testing. Additional details on specific tests and results are discussed in Section 3.

On completion of the laboratory testing programme, offshore field descriptions were reviewed and updated as per the onshore test results. An N_{kt} assessment was performed to investigate whether a more accurate range of values could be applied to the CPT data than what was used during fieldwork. Geological environment and age were assigned to identified units and geological cross sections were produced to show the lateral variability of the units across Bornholm I and II. Additional details on field geology are presented in Section 8.

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Symbols

%	Percent
°	Degrees
µm	Micrometres
A_c	Projected area of the cone
B_q	Pore Pressure Ratio
cm	Centimetres
C_u	Degree of Sorting
d	Depth of borehole below sealevel
D_{10}	10% of soil particles are finer than this size
D_{60}	60% of soil particles are finer than this size
D_r	Relative density / Equivalent relative density
F_s	Sleeve Friction
g	grams
G0	Initial Sear modules
I_L	Liquidity Index
I_P	Plasticity Index
K_0	Coefficient of lateral earth pressure
kN	Kilo Newton
kN/m^2	Kilonewton per square meter
kPa	Kilo Pascal
L_l	Liquid Limit
m	Metre
m3	Mass of the crucible and oven-dry soil specimen
m4	Mass of the crucible and specimen after ignition
mc	Mass of the crucible
ml	Millilitres
mm	Millimetre
MPa	Mega Pascal
N_{kt}	Cone factor
ºC	Degrees Celsius
P_L	Plastic Limit
q_c	Measured cone resistance
q_{net}	Net cone resistance
q_t	Corrected total cone resistance
R_f	Friction ratio
s_u	Undrained shear strength (intact)
u_0	Hydrostatic pore pressure
u_2	Pore water pressure measured behind the tip shoulder
w	Water content
Wi	Liquid Limit
Wp	Plastic Limit
z	Depth of cone tip below bottom of borehole
α	Cone area ratio
γw	Unit weight of water
Δu	Pore pressure in excess of hydrostatic
σ_1-σ_3	Deviatory stress
σ_m’	Estimated mean effective stress at test depth
σ_{v0}	Total overburden stress

LIST OF ABBREVIATIONS

Abbreviations

A	Axial
API	American Petroleum Institute
ASR	Average Stress Ratio
ASTM	American Society for Testing and Materials
B	Bag
BS	British Standard
BSB	Below Seabed
CAU	Consolidated Anisotropic Undrained
CAUc	Anisotropically Consolidated Undrained Triaxial in Compression
CAUcyc	Cyclic Anisotropically Consolidated Undrained Triaxial in Compression
CH	Clay of high plasticity
CIDc	Isotropically Consolidated Drained Triaxial in Compression
CIUc	Isotropically Consolidated Undrained Triaxial in Compression
CL	Clay
CM	Clay of intermediate plasticity
CO_2	Carbon Dioxide
CPT	Cone Penetration test
CPTU	Cone Penetration Test (with pore pressure)
CSR	Cyclic Stress Ratio
CV	Clay of very high plasticity
D	Diametrical
DGN	Differential Global Navigation Satellite
DSS	Direct Simple Shear
Dth	Down the hole
ETRS	European Terrestrial Reference System
Ex	Example
HCl	Hydrochloric acid
HF	Hydrofluoric acid
HNO_3	Nitric acid
I	Irregular Block/Lump
I2	I2 Analytical

ISO	International Standards Organisation
ISRM	International Society for Rock Mechanics
LoI	Loss of Ignition
MH	Silt of high plasticity
ML	Silt of low plasticity
MLV	Motorised Lab Vane
OCR	Overconsolidation Ratio
OH	Gyttja of high plasticity
OL	Gyttja of low plasticity
OWF	Offshore Wind Farm
P-S	Compression-Shear Wave
P	Push Number
PSD	Particle Size Distribution
PWP	Pore Water Pressure
Q	Quart
RGL	Robertson Geologging Ltd
SBE	Single Beam Echo Sounder
SCPTU	Seismic Cone Penetration Test (with pore pressure)
Stbd	Starboard
T	Tube
UCS	Unconfined Compressive Strength
USC	Unified Soil Classification
UTM	Universal Transverse Mercator
UUT	Unconsolidated Undrained Triaxial
WISON	Wireline Sounding
$ZnCl_2$	Zinc Chloride

VOLUME II: MEASURED AND DERIVED GEOTECHNICAL PARAMETERS AND FINAL RESULTS

1 Scope of Project

1.1 General

Energinet Eltransmisson A/S (Energinet) commissioned Gardline Limited (Gardline) to support the development of a new offshore wind farm (OWF) through the acquisition of geotechnical data which will inform the basis for evaluation of methods for wind turbine foundation design and installation. The objective will be met through acquiring ground information through both Seabed CPT data and Down the Hole (DtH) Sampling, Coring and CPTU. PS logging was also conducted at selected locations.

The objectives of the geotechnical investigation were to acquire geotechnical *in situ* data, samples and information as basis for evaluation of preliminary wind turbine foundation design and installation, as well as preliminary design of wind turbine configuration. This will be supported through the completion of an onshore laboratory testing programme and the results reported to inform the feasibility of achieving the safe installation and operation of the Bornholm I and Bornholm II OWF.

The geotechnical survey comprised of seabed CPTUs (both 200kN rig and 35kN Neptune 5000 rig), composite boreholes and PS logging. The composite boreholes comprised of sampling, coring and CPTU utilising a downhole WISON system as well as Geobore-S and PQ3 coring techniques. SCPTU tests were originally proposed to be conducted in seabed mode. However, no SCPTU data was acquired due to the soft nature of the ground conditions, excessive sinkage of the 200kN rig and the inability to acquire SCPTU data using the Neptune 5000.

The termination criteria for each location, as laid out in the scope of work, is provided in Table 1.1. Composite boreholes were sample-only from mudline to the bottom depth of the corresponding seabed CPTU location, then utilised a combination of CPTU, associated over-sampling, and undisturbed sampling techniques until rock was identified or target depth was reached. Where rock was encountered, coring was performed until target depth.

Table 1.1 – Scope of Work Termination Criteria

Acquisition Type	Target Depth(m)
Seabed CPTU – continuous	70m or refusal
Composite Sampling, Coring and CPTU Boreholes with PS Logging in Selected Locations at 1m Intervals	70m*
*Geobore-S coring operations limited to 100m (combined water and borehole depth)	

The water depths were obtained by SBES (Single Beam Echo Sounder). Depths are to the vertical datum MSL (Mean Sea Level) in metres. Additional information on the Geodetics can be found in Section 7. The water depth at each location is presented on logs in APPENDIX 2.

Soil samples and rock cores acquired in the field were then subject to a comprehensive laboratory testing programme. This included classification, chemical, consolidation, total stress and effective stress testing. Tests were scheduled to develop a broad view of the properties of the soils and rocks encountered in Bornholm I and Bornholm II. Further details on the laboratory testing programme can be found in Section 3.

Digital data was provided for this project in the form of Excel Spreadsheets and AGS4.1 data. The files provided separately to this report are as follows:

- AGS4.1 data file for the project, including all relevant information on fieldwork operations and laboratory testing (Excludes CAUcy AGS4.1 data)
- AGS4.1 data for the cyclic triaxial testing programme (In Excel format)
- Digital data file for DSS testing (In Excel format)
- Digital data file for PS Logging (In Excel format)

2 Geotechnical Descriptions and Soil Profiles

2.1 General

This chapter presents the interpretative composite boreholes, and seabed mode CPTU with test results from the offshore and onshore work. Logs are presented 10.00m per page with each log including interpretations of CPTU, sample and core data, where performed. Coordinates and water depths along with dates of each location are stated on each log. Water depth measurements were reduced to MSL in metres.

All descriptions were reviewed and updated where required following the completion of the onshore laboratory testing programme. When updating the unit descriptions for shear strength using the data from the onshore tests, some DSS test results showed significantly smaller values than effective stress triaxial tests conducted in the same geological unit. In these instances, the effective stress triaxial results were deemed to be more representative of the soil strength as a much larger sample is tested under effective stress conditions.

The push sample column as presented on the hybrid logs and combined logs presents different shades to illustrate sample recoveries; white is no recovery, light grey is penetration and dark grey is recovery.

Geotechnical logs can be found in APPENDIX 2.

2.2 Geotechnical Profiles

Table 2.1 summarises the parameters presented on the logs and indicated where parameters were measured or derived. Derived CPT parameters are discussed in further detail in Section 4.

Table 2.1 – Parameters Presented on Logs

Data Type	Measured OR Derived	Symbol	Data Units	Log
Soil Description	Measured	-	-	All
Corrected Cone Tip Resistance	Derived	q_t	MPa	CPTU, Composite
Net Cone Tip Resistance	Derived	q_{net}	MPa	CPTU Derived
Sleeve Friction	Measured	f_s	MPa	CPTU, Composite
Pore Water Pressure	Measured	u_2	kPa	CPTU, Composite
Ambient Pore Water Pressure	Derived	u_0	kPa	CPTU, Composite
Friction Ratio	Derived	R_f	%	CPTU
Pore Pressure Ratio	Derived	B_q	%	CPTU
Undrained Shear Strength	Both	s_u	kPa	All
Relative Density	Derived	D_r	%	CPTU, Composite
Water Content	Measured	-	%	Sample, Composite
Density	Measured	-	Mg/m^3	Sample, Composite

Derived undrained shear strength from CPT data is calculated using one of two sets of N_{kt} parameters; 12.5-16.5 and 15.0-20.0. The selection of which N_{kt} set to apply to the CPT data is dependent on the material strength. This is discussed in further detail in Section 4. Where the 12.5-16.5 range is used, the derived undrained shear strength is presented as lime green on the logs. Where the 15.0-20.0 range is used, the derived undrained shear strength is presented as red on the logs.

2.3 Soil/Rock Layering

Layer descriptions were based on visual sample descriptions carried out offshore and, *in situ* test results. The presented descriptions were then revised upon completion of the laboratory testing programme onshore. Unit boundaries were also reviewed by comparing the revised descriptions with nearby units at the same location as well as the unit descriptions from other nearby locations.

The shear strength for each fine-grained soil layer is initially classified based on shear strength values from offshore test results and revised from the results of the onshore laboratory tests. The compressive strength of each rock layer is initially classified in the field using a geological hammer and then revised using the data from the onshore Unconfined Compressive Strength and Point Load test results.

3 Laboratory Testing Programme

3.1 General

The objective of the laboratory testing programme was to evaluate the pertinent physical and mechanical characteristics of the soils and rocks encountered at the site. This section of the report discusses the laboratory testing programme performed. Tests were performed in accordance with standards outlined by Energinet Eltransmission A/S and are summarised in Table 3.1.

Onshore laboratory test results are considered to be of high quality and representative of the soil and rock materials encountered in the Bornholm I and Bornholm II fields. Any issues with specific tests are discussed in greater detail within the relevant subsections of this chapter, and comments provided within test enclosures.

Table 3.1 – Laboratory Testing Standards

Laboratory Test Type	Standard
Geological Description	Danish Geotechnical Society Bulletin 1, ISO 14688, ISO 14689
Water Content	ISO 17892-1
Saturated Water Content	BS 1377-2
Bulk and Dry Density	ISO 17892-2
Bulk and Dry Density (Immersion Method)	ISO 17892-2
Particle Density	ISO 17892-3
Particle Size Distribution (Wet Sieve)	ISO 17892-4
Particle Size Distribution (Hydrometer)	ISO 17892-4
Incremental Loading Oedometer	ISO 17892-5
Unconfined Compressive Strength (UCS)	ISO 17892-7
Point Load	ISRM Point Load Method
Unconsolidated Undrained Triaxial (UUT)	ISO 17892-8
Isotropically Consolidated Undrained Triaxial (CIUc)	ISO 17892-9
Isotropically Consolidated Drained Triaxial (CIDc)	ISO 17892-9
Anisotripocally Consolidated Undrained Triaxial (CAUc)	ISO 17892-9
Atterberg Limits (4 Point Method)	ISO 17892-12
Pocket Penetrometer	ISO 19901-8
Loss on Ignition	BS 1377-3
Acid Soluble Sulphate	BS 1377-3
Acid Soluble Chloride	BS 1377-3
Torvane	BS 1377-7
Motorised Lab Vane (MLV)	BS 1377-7
Fallcone	ISO 17892-6
Rapid Determination of Carbonate Content	ASTM D4373-21
Cyclic Anisotropically Consolidated Undrained Triaxial (CAUcyc)	ASTM D5311
Thermal Conductivity	ASTM D5334-14
Direct Simple Shear (DSS)	ASTM D6528
Angularity of Grains	In-house method
Determination of Maximum and Minimum Density of Sands	Geolabs/NGI Method
Palynology	Network Stratigraphic Method
Micropalaeontology	Network Stratigraphic Method

A summary of completed onshore laboratory tests and the laboratory they were completed in are presented in Table 3.2. Numbers presented in this table represent the tests that were scheduled for the project along with how many of these were completed and how many were cancelled. Reasons for test cancellation were considered when updating unit descriptions. All cancelled tests were reviewed for alternative samples. However, if a test was cancelled due to the material type, a replacement test was not always possible to achieve.

Table 3.2 – Onshore Laboratory Testing Schedule

Classification Test Type	Lab	Scheduled	Completed	Cancelled
Sample Photographs	Gardline	209	205	4
Water Content	Gardline	140	154*	2
Saturated Water Content	Geolabs	11	7	4
Bulk and Dry Density	Gardline	139	121*	16
Bulk and Dry Density (Immersion)	Geolabs	121	101	23
Particle Density	Gardline	220	215	5
Atterberg Limits (4 Point Method)	Gardline	179	171	8
Particle Size Distribution (Wet Sieve)	Gardline	263	263	0
Particle Size Distribution (Hydrometer)	Gardline	236	232	4
Angularity	Gardline	43	43	0
Maximum and Minimum Dry Density	Gardline	51	16	35
Carbonate Content	Gardline	67	67	0
Acid Soluble Sulphate	i2	67	67	0
Loss on Ignition	Gardline	65	65	0
Thermal Conductivity	Gardline	24	16	8
Acid Soluble Chloride	GEOLABS	66	66	0
Oedometer	Gardline	52	45	7
MLV	Gardline	8	3	5
MLV Remoulded	Gardline	8	3	5
Fall Cone**	Gardline	0	7	0
UUT	Gardline	114	97	17
UCS (Stress-Strain)	GEOLABS	77	49	28
UCS	GEOLABS	50	24	26
Point Load***	GEOLABS/ Gardline	125	121	8
CIUc	GEO	22	22	0
CIDc	GEO	27	27	0
CAUc	GEO	30	29	1
CAUcyc	GEO	9	9	0
DSS	GEO	17	17	0
Palynology	Network Stratigraphic	3	3	0
Micropalaeontology	Network Stratigraphic	2	2	0

*Water content and density test completed numbers include additional tests for the Shelby Tube samples extruded onshore
**Fall cone Testing was not scheduled but conducted on cancelled UUT tests where possible
***Point Load test numbers include tests conducted on cancelled UCS tests

A summary of completed offshore laboratory tests across each vessel are presented in Table 3.3 and Table 3.4 below.

Table 3.3 – Offshore Laboratory Testing Schedule – M.V. Kommandor Susan

Classification Test Type	Completed
Sample Photographs	2313
Water Content*	1063
Bulk & Dry Density	506
Bulk Density	213
Pocket Penetrometer	266
Torvane	331
Motorised Lab Vane	131
*Water content numbers include water contents from density ring tests	

Table 3.4 – Offshore Laboratory Testing Schedule – M.V. Ocean Vantage

Classification Test Type	Completed
Sample Photographs	287
Water Content*	134
Bulk & Dry Density	90
Bulk Density	0
Pocket Penetrometer	58
Torvane	57
Motorised Lab Vane	18
*Water content numbers include water contents from density ring tests	

Due to the soft nature of the shallow soils encountered across Bornholm I and Bornholm II, many samples were retained in their Shelby Tubes for onshore extrusion, photography, logging and testing. This was done to minimise sample disturbance and ensure high quality test results. In total, 188 samples were extruded at Gardline's onshore laboratory.

3.2 Soil and Rock Description

Descriptive terms are based on "A guide to engineering geological soil description" - Danish Geological Society, Bulletin 1 and both ISO 14688 and ISO 14689 and are described in the following order:

- MAIN SOIL/ROCK TYPE
- Degree of induration
- Grain size and degree of sorting (SAND) / plasticity (CLAY)
- Minor components
- Structures
- Colours
- Mineralogy
- Carbonate content
- Colloquial names (if known)
- (Relative density) / (Shear strength) / (Compressive strength)
- Depositional Environment
- Age

3.2.1 Main Soil/Rock Type

The classification of soils is usually based on grain size, induration, sorting and plasticity. The classification of rocks is based on engineering experience, physical and structural properties, compressive strength, induration and basic field tests such as the rock acid test. Quality of the rock cores is defined by the total core recovery, solid core recovery and rock quality designation; these are not included in the rock descriptions but are presented on the borehole logs. For clarity, the main soil or rock type is always written in capital letters.

3.2.2 Degree of Induration

The degree of induration for soils and rocks is based on the descriptive terms in the Danish Geotechnical Bulletin and is presented in Table 3.5. Soils are defined by degrees of induration H1 and H2, where higher degrees of induration are used to indicate that the material is rock.

Table 3.5 – Degree of Induration

Symbol	Term	Description
H1	Unlithified	The material can easily be formed by hand. Grainy material will fall apart when dry.
H2	Slightly Indurated	The material can easily be cut with a knife and can be scratched with a fingernail. Individual grains can be picked out with the fingers when the material is grainy.
H3	Indurated	The material can be cut with a knife but cannot be scratched with a fingernail. Individual grains can be picked out with a knife when the material is grainy.
H4	Strongly Indurated	The material can be scratched with a knife. Individual grains do not come out with a knife. Fractures will follow grain surfaces.
H5	Very Strongly Indurated	The material cannot be scratched with a knife. Cracks and fracture surfaces will go through individual grains in grainy material.

3.2.3 Particle Size, Degree of Sorting and Plasticity of Soils

The basic soil types as defined by particle size analysis are as follows:

GRAVEL	Coarse	20.0mm to 60.0mm
	Medium	6.0mm to 20.0mm
	Fine	2.0mm to 6.0mm
SAND	Coarse	0.60mm to 2.0mm
	Medium	0.2mm to 0.60mm
	Fine	0.060mm to 0.2mm
SILT	Coarse	0.02mm to 0.060mm
	Medium	0.0060mm to 0.02mm
	Fine	0.002mm to 0.0060mm
CLAY		Less than 0.002mm

The soil descriptions presented were derived from visual description and reviewed once the particle size analysis results became available. Please note that where the term “Fines” is presented is this report, this relates to the total percentage of SILT and CLAY particles in soil material. They are considered as fines as the individual particles cannot be seen by eyesight alone.

The degree of sorting (C_u) is defined after particle size grading analysis using the expression d_{60}/d_{10}; Table 3.6 below presents the descriptive terms using on the logs based on the Danish Geotechnical Bulletin.

Table 3.6 – Degree of Sorting

Degree of Sorting	C_u
Well Sorted	$C_u < 2$
Sorted	$2 < C_u < 3.5$
Poorly Sorted	$3.5 < C_u < 7$
Unsorted	$C_u > 7$

A glacial TILL deposit must always be expected to contain all grainsize fractions and to be unsorted. Therefore, both grainsize and sorting are omitted when describing glacial TILL. Classification of glacial TILL deposits are based on the Danish Geotechnical Bulletin summarised in Table 3.7.

Table 3.7 – Classification of glacial TILL deposits

Grain Fraction	CLAY (%)	SILT (%)	SAND (%)	$I_p = W_l - W_p$ (%)
GRAVEL TILL	<12		>25	<4
SAND TILL	<12		>45-50	<4
SILT TILL	<12	>45-50		<4
CLAY TILL,				
(Very silty)	12-15			4-7
(Very sandy)	12-15			4-7
(Sandy (“normal”))	12-15			7-10
(Medium plasticity)	>20			>10
(High plasticity)	>30			>25

The identification and description of fine-grained soils during offshore operations is based on a set of hand tests including plasticity and dilatancy. Descriptions were reviewed upon completion of classification tests.

Table 3.8 below outlines the criteria used to determine the degree of plasticity as described in the Danish Geotechnical Bulletin.

Table 3.8 – Degree of Plasticity

Term	w_L%	I_P%	Clay Percentage (%)	USC-System
Clay, very fat	>80	>50		CV
Clay, fat	50-80	25-50		CH
Clay, lean	30-50	10-25		CM
Clay, silty / sandy	<30	7-10	15-20	CL
Clay, very silty / sandy	<30	4-7	10-15	CL
Silt, very clayey		4-7	<10	ML
Silt, clayey / sandy		<4	<10	ML

OL: Gyttja of Low plasticity
OH: Gyttja of High plasticity
ML: Silt of low plasticity
MH: Silt of high plasticity
CV: Clay of very high plasticity
CH: Clay of high plasticity
CM: Clay of intermediate plasticity
CL: Clay of low plasticity

3.2.4 Secondary Components

The description of secondary constituents was performed offshore by visual observation and later reviewed after completion of the particle size distribution results onshore.

Secondary soil constituents within a fine soil are classified based on the descriptive terms used in the Danish Geotechnical Bulletin and are summarised in Table 3.9.

Table 3.9 – Secondary Constituent Classification (Fine Soils)

Term	Principal Soil Type	Secondary Constituent
Slightly sandy / slightly gravelly	SILT or CLAY	<10%
Sandy / gravelly	SILT or CLAY	10 - 25%
Very sandy / very gravelly	SILT or CLAY	>25%

Secondary soil constituents within a coarse soil are classified as summarised in Table 3.10.

Table 3.10 – Secondary Constituent Classification (Coarse Soils)

Term	Principal Soil Type	Secondary Constituent
Slightly clayey / slightly silty	SAND or GRAVEL	<5%
Clayey / silty	SAND or GRAVEL	5 - 10%
Very clayey / very silty	SAND or GRAVEL	>10%

In addition to describing secondary soil constituents, shells are described. The terms used are outlined in Table 3.11 below and are based on the descriptive terms used in the Danish Geotechnical Bulletin.

Table 3.11 – Shell Classification

Shell Classification	Description
Shells	Indicating intact or almost intact shells
Shell Pieces	Indicating pieces which can easily be identified by an expert
Shell Fragments	Indicating that the shell fragments are of a size rendering a determination impossible without use of a microscope

3.2.5 Structures

Structures are divided into sedimentary and tectonic. Sedimentary structures include layering, laminations, schliering, crossbedding and anything formed by biological activity. Definition of beds and laminations were based on ISO 14688 as presented in Table 3.12.

Table 3.12 – Bedding Classification

Mean Thickness/Spacing (mm)	Bedding Thickness Term	Bedding Spacing Term
<6	Thin Lamination	Extremely Closely Spaced
6 – 20	Thick Lamination	
20 – 60	Very Thin Bed	Very Closely Spaced
60 – 200	Thin Bed	Closely Spaced
200 – 600	Medium Bed	Medium Spaced
600 – 2000	Thick Bed	Widely Spaced
>2000	Very Thick Bed	Very Widely Spaced

Tectonic structures include folds, fractures, slickensides and faults. Natural rock fractures were classified using the descriptive terms in the Danish Geotechnical Bulletin as presented in Table 3.13.

Table 3.13 – Fracture Classification

Symbol	Term	Description
S1	Unfractured	No fractures observed
S2	Slightly Fractured	Distance between fractures is greater than 10 cm. No vertical fractures.
S3	Fractured	Distance between fractures between 6 and 10 cm.
S4	Very Fractured	Distance between fractures between 2 and 6 cm.
S5	Crushed, Blocky	Distance between fractures less than 2 cm.

3.2.6 Colour

A Munsell colour chart was used for reference when describing the colour of the sample.

3.2.7 Mineralogy

Where contrasting mineralogy was observed in the sample it was noted, such as pockets of glauconite or pyritised burrows.

3.2.8 Carbonate Content

Carbonate content was tested offshore by placing a drop of dilute Hydrochloric Acid (HCl) on the sample and noting the strength of the reaction that occurred.

3.2.9 Colloquial Soil Type Names

Colloquial names were not included in soil descriptions.

3.2.10 Strength Classification

According to ISO 14688 and ISO 14689, fine grained soils are described using shear strength and rocks by unconfined compressive strength, as shown in Table 3.14 and Table 3.15 respectively. Strengths are initially obtained in the field from index shear strength tests on fine grained soils and geological hammer blows on rocks and revised upon the completion of the onshore laboratory testing programme. Relative densities of granular soils were derived from CPTU data as discussed in Section 4.

Table 3.14 – Soil Strength Classification

Undrained Shear Strength of Clays	Undrained Shear Strength (kPa)
Extremely low	0 – 10
Very low	10 - 20
Low	20 – 40
Medium	40 – 75
High	75 – 150
Very high	150 – 300
Extremely high	>300

Table 3.15 – Rock Strength Classification

Unconfined Compressive Strength of Rocks	Unconfined Compressive Strength (MPa)	Qualitative Interpretation of UCS (Geological Hammer)
Extremely Weak	0.6 – 1.0	Gravel size lumps crush between finger and thumb. Indented by thumbnail.
Very Weak	1 – 5	Crumbles under firm hammer blows. Can be peeled by knife.
Weak	5 – 25	Can be peeled with difficulty. Point of hammer makes shallow indents.
Medium Strong	25 – 50	Cannot be peeled with knife, fractures with single blow of hammer.
Strong	50 – 100	Rock broken by more than one hammer blow.

Unconfined Compressive Strength of Rocks	Unconfined Compressive Strength (MPa)	Qualitative Interpretation of UCS (Geological Hammer)
Very Strong	100 – 250	Requires many hammer blows to break specimen.
Extremely Strong	>250	Rings on hammer blows. Only chipped with geological hammer.

3.2.11 Depositional Environment

The grain size, degree of sorting, strength, sedimentary structures, shells, fossils and presence of organic material were used to determine the depositional environment of soils.

3.2.12 Age

The geological age of sediments was determined by cross referencing identified depositional environments with geophysical data and results from the Network Stratigraphic palynology and micropalaeontology testing. Geological environment and age are further discussed in Section 8.

3.3 Classification Laboratory Test Results

Basic index laboratory tests were performed offshore during operations in the vessel soil laboratory. Testing included soil and rock descriptions, colour identification, natural moisture contents and bulk and dry densities.

Further classification testing was performed onshore based on the laboratory testing schedule agreed between Energinet and Gardline. This included, but is not limited to, water content, saturated moisture content, bulk and dry density, density by immersion, maximum and minimum density, particle density, particle size distribution and Atterberg limits. All classification tests were carried out at Gardline's onshore laboratory, excluding saturated moisture contents of chalk and density by immersion testing; these were carried out by Geolabs.

Offshore and onshore test results are presented in composite borehole logs in APPENDIX 2. Summary tables are presented in APPENDIX 7.

3.3.1 Water Content, Bulk Density and Dry Density

Water content, bulk density, and dry density tests were performed on all representative soil samples and as part of the advanced laboratory testing schedule.

Bulk densities of soil samples were measured by weighing samples of known volume immediately following sample extrusion. Samples were then placed in an oven for a set time before being allowed to cool and their dry weight taken.

Next, the samples are dried until they are at a constant mass. Constant mass is defined as the point in which there is less than 0.1% change in mass of the dry soil, when dried for at least one more hour. If the change in mass of the sample exceeds 0.1% then the sample is dried in the oven for a further hour, and the weighing process repeated until constant mass is achieved.

Water contents (without density) went through the same process. However, the volume of the soil was not known.

Density values were consistent across the site and generally show good repeatability. Water content values correlate well between locations. Water content and bulk and dry density values presented in this report are measured values and no corrections have been applied.

Moisture content testing was carried out in accordance with ISO 17892-1. Bulk and dry density testing was carried out in accordance with ISO 17892-2.

3.3.2 Saturated Moisture Content and Density by Immersion

Saturated moisture content and density by immersion testing was carried out onshore on representative chalk and rock samples respectively. These were determined by coating the samples in paraffin wax and suspending the sample in water to record the apparent mass. The mass of the samples before and after coating in wax are also recorded. Moistures were then determined by oven-drying the samples after the water suspension stage.

Density by immersion testing was carried out in accordance with ISO 17892-2. Saturated moisture content testing was carried out in accordance with BS1377-2.

3.3.3 Particle Density

Particle density tests were completed onshore as scheduled in the laboratory testing program.

Approximately 100g of sample material was oven-dried to a constant mass at a temperature between 105° and 110°C. The dried sample was grinded with a pestle and mortar, if required, to allow the material to pass through a 4mm sieve. The sample was then riffled to obtain at least two specimens with at least 10g of mass, which was then oven-dried and cooled in a desiccator.

The dry soil specimen is then placed in a pyncnometer, which is then filled with de-aired water; a vacuum is used to ensure that all air is removed from the equipment. The pyncnometer is then placed in a water bath and left to allow the temperature of the sample and de-aired water to reach equilibrium.

The mass of the pyncnometer is recorded for all stages of the test. The particle density of the soil sample is the ratio between the mass of the dry mineral particles and the mass of distilled water displaced by the dry mineral particles. The fluid pycnometer method was used for all particle density tests.

Testing was carried out in accordance with ISO 17892-3.

3.3.4 Atterberg Limits

Atterberg limits were performed using the 4-point method to determine the liquid limit, plastic limit, plasticity index, liquidity index and activity index of a cohesive soil. These help to understand the behaviour of fine-grained sediments encountered during the geotechnical survey.

Water content (w), Plastic Limit (P_L), Liquid Limit (L_L), Plasticity Index (I_p), Liquidity Index (I_L) and Activity Index (AI) were determined for cohesive samples to provide classification information. In each case, the liquid limit test was performed by the cone penetrometer method using a 30 degrees, 80g cone. Where necessary, specimens were washed through a 400µm sieve to exclude oversized material.

The liquid limit is the water content at which a soil changes from the liquid to the plastic state. The plastic limit is the water content at which the soil ceases to be plastic when dried further.

Plastic Limit is determined by using 15-20g of soil paste, allowing the specimen to partially dry enough to be rolled into a ball. The ball is then rolled between the palms of the hands until the heat of the hands has dried the soil sufficiently for slight cracks to appear on the specimen's surface.

Next, the specimen is divided into two portions of about equal mass, one portion being divided into three sub-portions. Each sub-portion is moulded into an approximate 6mm diameter thread between the first finger and thumb of each hand. The thread is then placed onto a mixing plate and rolled further, maintaining even thickness across the thread until it reaches 3mm diameter. The process is repeated until the specimen crumbles.

Once each of the three sub-portions have crumbled, the pieces are placed in a container with lid. For the second portion the process is repeated, and all three sub-portions placed into a second container with lid. Each portion then has water content determined according to ISO 17892-1. Plastic limit is calculated by the average of the two water content determinations.

Values of the Plasticity index (I_P) and the Liquidity index (I_L) have been calculated for all fine grained soils. The liquidity index I_L is an index property that relates the natural water content of a fine grained soil to its respective liquid and plastic limits and is expressed as:

$$I_L = \frac{w - P_L}{I_P}$$

The Activity Index is principally dependent on the amount and the type of clay minerals and organic colloids present as well as on the electrolyte content of the pore water within the sample. Activity Index is calculated using the following formula:

$$I_A = \frac{(L_L - P_L)}{CF}$$

Where CF = is the dry mass of particles having an equivalent diameter less than 0,002 mm, divided by the dry mass of the specimen (or of the dry mass of the specimen after removal of the coarse fraction) expressed as a percentage.

Testing was carried out in accordance with ISO 17892-12.

3.3.5 Particle Size Distribution (PSD)

PSD tests were performed on samples as part of the onshore laboratory testing campaign to understand the composition and *in situ* behaviour of the identified soil units.

Soils consist of discrete particles varying in shape and sizes. The purpose of a particle size distribution is to group these particles into size ranges and determine the relevant proportions, by dry weight, of each size range.

Two separate and different procedures are used:

- Wet sieving – used to assess the coarse-grained particle sizes of gravel and sand

- Sedimentation by hydrometer – used for finer silt and clay particles

During sedimentation, a reagent of sodium hexametaphosphate solution is used to help disperse the soil particles; this is made up of 40g of sodium hexametaphosphate in 1 litre of distilled water.

Soils were also pre-treated for organic content. This involved burning off organic matter with Hydrogen Peroxide and then filtering under a vacuum before testing.

PSDs are considered to be of good quality and achieved repeatable results. They strongly correlate with the soil type and behavioural characteristics noted during the logging phase.

Testing was carried out in accordance with ISO 17892-4.

3.3.6 Maximum and Minimum Density

Maximum and minimum densities were conducted as part of the onshore laboratory testing campaign, and can be used to feed into testing specification for other tests (e.g. test requiring reconstitution of cohesionless material, advanced testing parameters).

Prior to testing, particles over 2mm were removed by dry sieving.

Minimum density was determined by weighing a cylinder of a known volume and placing a funnel into the centre of the cylinder, with the base of the funnel flat at the bottom. Dried material was poured into the funnel until a cone of sample formed at the top. The funnel was slowly raised at a consistent speed while the sample poured out and filled the cylinder. The funnel was raised in a manner that ensured the funnel was just above the sample that was piling up so there were no obstructions. This was done until a cone of sample formed above the top of the cylinder, and the funnel was then moved away. The top of the sample was struck off using a straight edge, and excess sample was brushed from the outside of the cylinder. The filled cylinder was then weighed. This process was repeated until five determinations were made.

Maximum density was determined by riffling two equal 500g specimens. 100mL of distilled water was poured into a cylinder with a known height and internal diameter, the cylinder was then vibrated at an amplitude of 0.60, and the first 500g sample poured into the cylinder using a funnel as it's vibrated. After this vibration period, a disc of a known thickness is placed on top of the specimen and the excess water siphoned off. The distance from the top of the disc to the top of the cylinder is measured, and a surcharge weight of 2745g is applied. The specimen is then vibrated at an amplitude of 0.50 for no more than 15 seconds. After vibrating, the surcharge is removed, and any excess distilled water is siphoned off. The distance between the top of the disc and the top of the cylinder is measured again. The material is then removed from the cylinder and dried to constant mass. This procedure is repeated for the second 500g specimen.

This method for determining maximum and minimum density of sands minimises risk of operator influence from the test. In vibrating the max density specimen with a set amplitude and set surcharge weight ensures that the compaction will be uniform across both tests, instead of relying on the operator to evenly distribute blows from a compaction rammer.

This method also allows for a smaller specimen size to be used than the minimum required mass of BS 1377 and ASTM methods which has been the cause for many test cancellations in the past.

This method also allows specimens with up to 12% fines to be tested, whereas BS 1377 only allows for 10%.

Testing was carried out in accordance with the NGI/Geolabs in-house methodology as outlined above.

3.3.7 Angularity

Angularity descriptions were undertaken within a selection of granular soil samples. Samples were washed through a 63-micron sieve and oven-dried before being placed on a black plate. Two high quality photographs were then captured at a 60x and 205x lens magnification so that the angularity and sphericity of the soil grains could be described.

In total forty-three angularity descriptions were conducted across the site. They indicate a wide array of sand angularity, from rounded to angular.

Angularity testing was carried out in accordance with Gardline's in-house methodology as outlined above.

3.3.8 Thermal Conductivity

This test method presents a procedure for determining the thermal conductivity of a soil using a transient heat method. Samples can be tested either intact or reconstituted. A thermal probe is inserted into the middle of a sample. After reaching temperature equilibrium with the sample, the thermal probe is set to emit heat at a level so that the overall temperature rise during the test doesn't exceed 3°C. Measurements are then taken over a period of at least 200 seconds to allow for sufficient time to reliably determine the thermal conductivity of the test sample.

Some issues were encountered on the reconstituted sand specimens where the target density for the sample was too dense for sufficient penetration of the thermal probe. In these instances, the density of the sample was reduced to be the densest state possible to allow the test to be conducted. Remarks are presented on the test enclosures in these instances.

Testing was carried out in accordance with ASTM D5334-14.

3.4 Total Stress Laboratory Test Results

Index soil strength tests were performed offshore and onshore as these provide a fast method for clarification of undrained shear strength in fine grained cohesive soils. Index shear strength tests used offshore included Pocket Penetrometer and Torvane testing apparatus. Unconsolidated Undrained Triaxial (UUT) tests were performed onshore to aid design as agreed by Energinet and Gardline. Laboratory Vane tests were conducted both offshore and onshore. All total stress tests were carried out by Gardline.

Shear strength results on the fine-grained units encountered generally correlate well with one another, though some variation is expected due to the varying failures each test induces on the sample.

Index shear strength results are on the sample logs in APPENDIX 2 and in the shear strength summary tables in APPENDIX 8.

3.4.1 Pocket Penetrometer

The pocket penetrometer is a small handheld device consisting of a steel cylindrical plunger or adaptor and calibrated compression spring located in a cylindrical housing. The plunger is forced to penetrate the soil sample a fixed distance into the soil in a push time of approximately 1 second. The compression of the pocket penetrometer spring is directly calibrated to indicate the undrained shear strength of the soil (s_u). At least three readings should be performed on a specimen and the average of these readings taken as the final measurement.

Three different adapters for the pocket penetrometer were made available to facilitate measurements of very high to extremely high strength soils, with a maximum capacity of 1200kPa. Each adapter has a specified factor used to calculate the undrained shear strength.

Testing was carried out in accordance with ISO 19901-8.

3.4.2 Torvane

The Torvane is a small hand-operated device consisting of a plastic disc with thin, radial vanes projecting from one face. The Torvane is pressed against a flat surface of the soil until the vanes are fully embedded and is rotated through a torsion spring until the soil is sheared. The device is calibrated to indicate shear strength of the soil directly from the rotation of the torsion spring. Torvane testing conducted in the field was also used to determine whether to retain samples in their Shelby Tubes for onshore extrusion.

Testing was carried out in accordance with BS 1377-7.

3.4.3 Motorised Laboratory Vane

The miniature laboratory vane comprises a four bladed steel cruciform vane mounted on a rod. Typical vane dimensions are 12.7mm^2 and 25.4mm^2. Rotation of the vane is provided by a motor applying torque via worm and pinion drive with a scale graduated in 1° intervals for measuring angular rotation of the vane relative to the soil in which it is placed. A calibrated open coil torsion spring is used to increase torque with rotation. Shear is determined by the degree of rotation achieved after sufficient torque has been acquired to shear the vane within the sample.

For residual tests, the vane is rotated in the test position for at least two full turns of the blade by hand, and the spring tension is released. The sample is then sheared again in the same manner as the undisturbed test.

Residual tests provide a residual strength, and are therefore sensitivity is not determined. With more competent soils, it is often the case that after shearing and performing the residual steps, most of the soil where the vane has rotated has been displaced, so when the residual test is performed the vane is not fully in contact with soil.

Where samples were retained in their Shelby Tubes, lab vane testing was conducted on the ends of the sample where possible.

Testing was carried out in accordance with BS 1377-7.

3.4.4 Unconsolidated Undrained Triaxial (UUT)

The UUT test is used to determine shear strength and stiffness of a soil for design.

In the UUT the test specimen is encapsulated in a latex rubber membrane and subjected to a confining pressure to replicate *in situ* ground conditions. The soil specimen is then loaded axially in a load frame at a constant rate of strain; typically, in the order of 1-2% per minute until the specimen fails. No drainage is allowed at any stage of the test. The undrained shear strength of the soil, s_u is half of the deviator stress at failure:

$$s_u = \frac{\sigma_1 - \sigma_3}{2}$$

Where σ_1-σ_3 is the maximum deviator stress (kN/m^2).

Triaxial summaries and plots are presented in APPENDIX 10. Testing was carried out in accordance with ISO 17892-8.

3.4.5 Fall Cone

The fall cone test uses four small differently weighted cones held in a stationary position by the apparatus which is lowered up or down to ensure the cone only just touches the top of the soil sample. The cone is then dropped freely under its own weight in a vertical direction into the soils sample and its penetration marked not more than 2s after cone release. The test is carried out three times on the same sample and average penetration taken. Penetrations are read to the nearest 0.25mm and must be within 4 to 20mm. If values are outside this range a more suited cone to the soil conditions should be used.

Undrained shear strengths are calculated using a set of conversion charts or calibration factors depending on the penetration value and cone used.

Seven onshore Fall Cone tests were conducted on samples that were too soft to conduct UUT testing.

Testing was carried out in accordance with ISO 17892-6.

3.5 Chemical Tests

Carbonate content, mass loss on ignition, acid soluble sulphate and acid soluble chloride testing were conducted as part of the onshore testing programme. Carbonate content and loss on ignition testing was carried out at Gardline's onshore laboratory. Acid soluble sulphate testing was caried out by I2 and acid soluble chloride testing by Geolabs.

A summary of all chemical tests is presented in APPENDIX 11.

3.5.1 Carbonate Content

The test procedure is a gasometric method that utilises a simple portable apparatus. The carbonate content of soil is determined by treating a dried soil specimen with hydrochloric acid (HCl) in an enclosed reaction cylinder (reactor). Carbon dioxide (CO_2) gas is exsolved during the reaction between the acid and carbonate fraction of the specimen. The resulting pressure generated in the closed reactor is proportional to the calcite equivalent of the specimen. This pressure is measured with a suitable pressure gauge, or equivalent pressure-measuring device, that is pre-calibrated with reagent-grade calcium carbonate.

It should be noted that the results of this test are calcite equivalent as different carbonate species will result in percentages greater than 100%. This test does not distinguish between the carbonate species and such determination must be made using quantitative chemical analysis methods such as atomic absorption.

Testing was carried out in accordance with ASTM D4373.

3.5.2 Loss on Ignition

Loss on ignition testing is used to determine the proportion, by mass, which is lost from a soil by ignition at a certain temperature.

The sample is dried to a constant mass in an oven at 50 ±2.5°C, cooled to room temperature in a desiccator and weighed. The sample is then passed through a 2mm sieve. Any particles that are retained (other than stones) are crushed until they pass through the 2mm sieve. The mass passing the 2mm sieve is then recorded to the nearest 0.1%.

The sample is then divided by successive riffling to produce a sample weighing more than 10g and then pulverized and passed through a 425μm sieve. The sample which passes the 425μm sieve is then riffled to obtain test specimens each weighing 5g. Each specimen is then dried in an oven at 50 ±2.5°C until the difference in successive weighing's, every 4hours, does not exceed 0.1% of the original mass of the sample. The sample is then cooled in a desiccator and weighed to 0.001g.

Next, the crucibles are placed in a furnace, ignited at 440 ±25°C for at least 3 hours, removed from the furnace and allowed to cool to room temperature in a desiccator. The crucible and its contents are then weighed to the nearest 0.001g.

To calculate mass loss on ignition as a percentage of the dry mass passing a 2mm sieve the following equation is used:

$$LoI = \frac{m_3 - m_4}{m_3 - m_c} x100\%$$

Where

- m_3 is the mass of the crucible and oven-dry soil specimen
- m_4 is the mass of the crucible and specimen after ignition
- m_c is the mass of the crucible

Testing was carried out in accordance with BS 1377-3.

3.5.3 Acid Soluble Sulphate

This method determines the acid soluble sulphate content of soils by adding excess HCl to test specimens to dissolve the acid soluble sulphate species into solution. The sulphate in solution is determined gravimetrically by adding barium chloride to the extract to precipitate the sulphate as barium sulphate.

Testing was carried out in accordance with BS 1377-3.

3.5.4 Acid Soluble Chloride

This method determines the acid soluble chloride content in soil by extraction with nitric acid. Silver nitrate is added and then followed by titration against thiocyanate.

Testing was carried out in accordance with BS 1377-3.

3.6 Consolidation Tests

3.6.1 Incremental Loading Oedometer

Oedometer tests were completed to characterise stress history, compressibility and flow behaviour during one-dimensional compression or swelling of cohesive soils. Oedometer tests were scheduled with five loading phases during consolidation.

Samples were laterally confined in an oedometer ring and an increment of load was applied to the submerged sample until primary consolidation was achieved. Increments of load were applied to each sample to determine the soil-strain behaviour. Additional loading steps were added to the test if the data acquired was insufficient for predicting the pre-consolidation pressure.

Priority tests were selected to provide reference pre-consolidation pressures for the scheduling of the CAU compression tests and later the cyclic CAU compression tests.

Incremental loading oedometer test enclosures are presented in APPENDIX 12.

Testing was carried out in accordance with ISO 17892-5.

3.7 Effective Stress Tests

The effective stress testing program comprised of Isotropically Consolidated Undrained Triaxial Compression (CIUc), Isotropically Consolidated Drained Triaxial Compression (CIDc), Anisotropically Consolidated Undrained Triaxial Compression (CAUc), Anisotropically Consolidated Undrained Triaxial Compression with Cyclic Loading (CAUcyc) and Direct Simple Shear (DSS) tests. All effective stress tests were conduced by GEO.

Effective stress results are presented in APPENDIX 13.

3.7.1 Isotropically Consolidated Undrained Triaxial Compression Tests

CIUc tests were scheduled to determine the isotropically consolidated undrained shear strength, and pore pressure change during shear, when a specimen is subjected to compression under a known effective stress.

The test is carried out in three stages: saturation, consolidation and compression. The saturation stage is used to ensure that all voids are filled with water before testing. The consolidation stage immediately follows and is used to bring the specimen to the specified state of effective stress used for carrying out the compression test. The compression stage involves shearing the sample at a constant rate of axial deformation while the cell pressure is maintained. No drainage is permitted and therefore the moisture content remains constant during compression.

From a set of tests, the undrained effective shear strength parameters at failure can be derived.

Testing was carried out in accordance with ISO 17892-9.

3.7.2 Isotropically Consolidated Drained Triaxial Compression Tests

CIDc tests were scheduled to determine the isotropically drained shear strength, and volume change characteristics during compression of a specimen from which the pore water can drain freely.

The test is carried out in three stages: saturation, consolidation and compression. The saturation stage is used to ensure that all voids are filled with water before testing. The consolidation stage immediately follows and is used to bring the specimen to the state of effective stress used for carrying out the compression test. The compression stage involves shearing the sample at a constant rate of axial deformation while the cell pressure is maintained.

Some issues were encountered when reconstituting the sand specimens to their specified target densities, where these were either too high or too low to achieve. In these cases, the samples were reconstituted to either the densest state or loosest state possible. The reconstitution procedure was not altered to achieve this. Remarks were left on the test enclosures in these instances.

For the test scheduled on BH-101 P06B1, there was insufficient material to conduct the test. To avoid test cancellation, material from BH-107 P19B1 was used to make up for the missing material as agreed between Gardline and Energinet.

Testing was carried out in accordance with ISO 17892-9.

3.7.3 Anisotropically Consolidated Undrained Triaxial Compression Tests

CAUc tests were scheduled to determine the undrained shear strength characteristics and stress-strain relationships. The tests were preloaded anisotropically to an estimate of the preconsolidation effective stresses, then unloaded back to the existing *in situ* effective stresses prior to shearing. In this way, the specimen is taken through a stress path mimicking its consolidation history and during which it is intended that any disturbance that the specimen may have experience during sampling, or stress relaxation from exhumation, may be ‘repaired’ prior to the shearing of the sample.

Data from either CPTU or oedometer testing was used to predict the K0 for scheduling the pressures for the CAU tests. In instances where the K0 calculation returned values greater than one, these were reduced to one and the test rescheduled as an isotropic undrained test.

Priority tests were selected to provide reference undrained shear strength values for the scheduling of the cyclic CAU compression tests.

Testing was carried out in accordance with ISO 17892-9.

3.7.4 Cyclic Anisotropically Consolidated Undrained Triaxial Compression Tests

CAUcyc tests were scheduled to determine the undrained shear strength characteristics and stress-strain relationships with cyclic loading. The tests were prepared and consolidated in the same manner as the CAUc tests. Cyclic loading generally causes an increase in the pore water pressure of the specimen, which results in a decrease in the effective stress and an increase in the cyclic axial deformation.

Pressures were calculated using q_{net} values from CPTU tests, reference values from nearby static tests, and CSR and ASR values as provided by Energinet. Tests were scheduled with a range of CSR and ASR values with the aim that the results of the nine scheduled tests can be used by future developers to create failure contour envelopes for design work.

Table 3.16 provides a summary of the test specifications for each location.

Table 3.16 – Cyclic CAUc Test Specifications

Test Specimen	Average Static Stress (kPa)	Cyclic Shear Stress (kPa)	Cyclic Average Strain (%)	CSR	ASR
BH-104 P05Q1	102	15.33	45.99	0.15	0.45
BH-108 P09Q2	78	35.15	0.00	0.45	0.00
BH-110 P07Q2	25	10.08	0.00	0.40	0.00
BH-201 P11Q1	57	19.95	0.00	0.35	0.00
BH-202 P44Q2	254	88.80	0.00	0.35	0.00
BH-203 P11Q3	94	47.15	47.15	0.50	0.50
BH-204_b P17Q1	389	155.76	155.76	0.40	0.40
BH-209 P09Q2	106	42.32	42.32	0.40	0.40
BH-217 P11Q2	290	144.85	0.00	0.50	0.00

The test scheduled on BH-110 P07Q1 was observed to swell during the saturation phase of the test. GEO increased the effective stresses on the specimen from 20kPa to 40kPa, at which point no more swelling was observed. To avoid swelling during the consolidation phase of the test, the consolidation pressures were slightly increased from what was initially scheduled. No further issues were observed for this test; relevant remarks are presented on the test enclosure.

The test scheduled on BH-201 P11Q1 was not anisotropically consolidated for long enough to allow the sample to reach its in-situ pressure conditions due to technician error. This may have some implications on the reliability of these test results and so the data for this test should be treated with caution. GEO however has confidence that these results are still suitable for use in design work.

Testing was carried out in accordance with ASTM D5311.

3.7.5 Monotonic Direct Simple Shear

Monotonic DSS tests were scheduled to determine the stress-strain-strength relationships for monotonic horizontal loading.

DSS tests were laterally confined using rings and consolidated in three stages to the vertical in situ stress. Samples were then monotonically sheared under constant volume conditions to determine the stress-strain behaviour of the soil.

The first two consolidation loading stages for the tests scheduled on BH-102 P03Q2 and BH-105 P04Q1 were lower than the initial seating pressure of the DSS plates. To avoid the risk of sample swelling during the first two consolidation loading stages, these two tests were consolidated under the final loading stage only, for an extended period of time as approved by Energinet. No significant impact on the results of these tests was observed by using this approach.

Testing was carried out in accordance with ASTM D6528-17.

Results are presented in APPENDIX 14.

3.8 Rock Test Results

Unconfined compressive strength and point load testing of rock samples was conducted as part of the onshore testing programme. All UCS testing was carried out by Geolabs. Point load testing was carried out by both Geolabs and Gardline.

A summary of all rock tests and relevant test enclosures are presented in APPENDIX 9.

3.8.1 Unconfined Compressive Strength (UCS)

UCS testing was scheduled to determine the unconfined compressive strength of selected intact rock core specimens.

The specimen is loaded axially in a load frame at atmospheric pressure and at a constant rate of strain; typically, in the order of 0.5-1% per minute until the specimen fails, in order to determine the stress-strain behaviour. Specific tests were also scheduled to present the stress-strain curves on the test enclosures to evaluate the Young's Modulus of the rock samples.

Testing was carried out in accordance with ISO 17892-7.

3.8.2 Point Load

Rock point load tests provide a fast method for index strength tests which can be used to classify and characterise the rock.

A section of rock is taken for testing at its natural moisture content; the shape/dimensions of the rock determine whether the test performed will be Diametral (D), Axial (A), or Block/Irregular Lump (I). The distance between platens before testing is measured by determining the height of the rock specimen in the orientation it is tested in. The test specimen is then placed between the platens of the apparatus, and after ensuring that the gauge is set to 0, a load is applied to the specimen steadily by manually pumping the apparatus until failure occurs. The peak load reading is then recorded, along with the distance between platens at failure. Failure is considered to be invalid if it occurs outside of the stated 10-60 seconds or if the generated fracture surface only passes through one of the two loading points in the testing apparatus.

Testing was carried out in accordance with the ISRM Point Load Method.

3.9 Other Test Results

3.9.1 Palynology and Micropalaeontology

Palynology and Micropalaeontology testing was carried out on selected samples to acquire information to support the interpretation of the geological ages across the Bornholm I and Bornholm II fields. Palynology involves the identification and analysis of terrestrial and marine taxa, whereas Micropalaeontology involves the analysis of miscellaneous microfossils.

Three organic-rich samples were selected across the sites to be used to support in defining geological age; two of these were in peat sediments and one was in a coal bed. The aim of this testing was to aid in developing the interpretation of the bedrock age. However, the results returned that all samples tested were glacially disturbed, with the coal sample possibly early cretaceous/Jurassic but reworked in glacially disturbed sediments.

Further testing is recommended to develop the analysis of these results.

The palynology and micropalaeontology report is presented in APPENDIX 15. Detailed procedures for these tests are described in this report.

Testing was carried out in accordance with Network Stratigraphic's in-house methodology.

4 CPTU Analysis

4.1 General

4.1.1 Downhole Composite CPTU Boreholes

Downhole CPTU data was acquired at 46 composite boreholes at 32 locations (including 14 bump-overs) utilising a WISON downhole system via an independent winch and umbilical cable in combination with the drill rig. Downhole CPTUs were carried out in accordance with the requirements of ISO 19901-8:2014. Tests were conducted with 10cm^2 and 5cm^2 cones calibrated in accordance with BS 22476-1:2012 and ISO 19901-8:2014.

4.1.2 Seabed CPTU Boreholes

In total, 39 seabed mode CPTUs were undertaken at 24 locations (including 15 retests) using Gardline's 200kN thrust capacity seabed CPTU rig system which allows the acquisition of high quality continuous CPTU data. 120 seabed mode CPTUs were undertaken at 63 locations (including 57 retests) using Gardline's 35kN thrust capacity seabed CPTU rig system (Neptune 5000) which allows the acquisition of high quality continuous CPTU data at softer seabed conditions. Additional details on the soft seabed conditions are discussed in the Field Operations Report.

Acquired CPTU data was processed using Gardline's proprietary TerraFusion software.

4.2 CPTU Results

In total 793 CPTUs were completed during the site investigation; of these, 635 were conducted in downhole mode and 158 in seabed mode. Cone offsets were taken on deck and at seabed/down the hole before and after each test for all CPTU tests. To enhance fieldwork progression of the seabed CPTs, the seabed frame was not always brought back up to deck after each test and instead was left hovering above the seabed while the vessel moved between locations. In these instances, the application classes of these tests was determined using the final deck to deck readings when the seabed frame was brought back up to deck. No more than three seabed CPTU tests were conducted before bringing the seabed frame back up to deck. Cone offsets are tabulated in CPTU Reference Readings for downhole CPTU in APPENDIX 3.

Throughout operations on BH-218, BH-218_a and BH-218_b pipe slippage has been observed in the CPTU data. Pipe slippage had been removed from the data, but the data is still deemed representative.

Table 4.1 and Table 4.2 below summarise the cone offset classifications of the downhole CPTU and seabed CPTU tests respectively. Where test data fell outside the required standard according to the shift engineer's judgement with reference to ISO 19901-8:2014, a cone change was conducted.

Table 4.1 – Downhole CPTU Application Class Summary

CPTU Application Class	Total No. Tests
Class 1	465
Class 2	137
Class 3	30
Out of Class	3
Total	635

Table 4.2 – Seabed CPTU Application Class Summary

CPTU Application Class	Total No. Tests
Class 1	106
Class 2	43
Class 3	8
Out of Class	1
Total	158

The SBF for the drill rig was fitted with base inclinometers, measuring the x-axis and y-axis of the rig, which was monitored throughout.

It should be noted that on occasion, CPTU results showed negative pore water pressure (PWP) when penetrating high to extremely high strength clays. This response is common in over-consolidated clays and is caused by cavitation during penetration. Cavitation can be easily identified in the CPTU records when negative PWP appears to reach a limit and flat-line, this limit corresponds to the cavitation pressure.

The minimum negative PWP reading that a cone can register will be a function of the cavitation pressure, which is given by the sum of atmospheric and hydrostatic pressures.

Minimum deck to deck values for each application class are defined in Table 4.3.

Table 4.3 – CPTU Application Class Criteria

Application Class	Measured Parameter	Allowable Min. Accuracy
1	Cone Resistance	35 kPa
	Sleeve Friction	5 kPa
	Pore Water Pressure	25 kPa
2	Cone Resistance	100 kPa
	Sleeve Friction	15 kPa
	Pore Water Pressure	50 kPa
3	Cone Resistance	200 kPa
	Sleeve Friction	25 kPa
	Pore Water Pressure	100 kPa

4.3 Presentation of Measured and Derived CPT Parameters

The CPTU data are presented in two sets of plots. The first set represents corrected measured parameters. These plots consist of:

- Cone resistance - q_c
- Sleeve friction - f_s
- Pore water pressure - u_2 (behind the tip)

The data acquired downhole was corrected for the error involved in zeroing the cone sensors at the borehole base:

$$u_2 = u_2^* + \gamma_w * d$$
$$q_c = q_c^* + d * \alpha * \gamma_w$$

Where

α	=	the ratio of the cone shaft to the area of the cone
u_2^* & q_c^*	=	the measured values of the downhole CPTU
d	=	depth of borehole below seabed
γ_w	=	unit weight of water
z	=	depth of cone tip below bottom of borehole

The second set represents derived parameters and contains the following calculated data:

- Corrected cone resistance - q_t
- Net cone resistance – q_{net}
- Friction Ratio - R_f
- Pore pressure ratio - B_q
- Hydrostatic Pore Pressure – u_0

The data for the derived plots were calculated using the following formulae:

$$q_t = q_c + \alpha^*\gamma_w{}^*d + (1-\alpha)(u_2 + \gamma_w{}^*d) = q_c + (1-\alpha)\, u_2 + \gamma_w{}^*d$$

$$R_f = f_s/q_t * 100\%$$

$$B_q = \Delta u/q_n$$

$$u0 = d * \gamma_w$$

where $q_{net} = q_t - \sigma_{v0}$

and $\Delta u = u_2 - \gamma_w * z$

α - Ratio of the cone shaft to the area of the cone tip

q_{net} - Net cone resistance

Δu - Pore pressure in excess of hydrostatic pressure

σ_{v0} - Total overburden pressure of overlying sediments

Measured and derived plots are presented for downhole mode and seabed mode CPTU in APPENDIX 3.

4.4 Undrained Shear Strength

Undrained shear strengths are only presented for fine grained undrained cohesive soils. Table 3.14 shows the strength descriptor used for CPTU and sample log interpretations. Where units of silt were encountered, care was taken to identify whether these units exhibit drained or undrained behaviour. Each unit was assessed on a case-by-case basis to provide the representative derived data.

Undrained shear strength (s_u) is calculated from CPT data using the following formula:

$$s_u = q_{net}/N_{kt}$$

where N_{kt} = Cone Factor

$q_{net} = q_t - s_{v0}$, the net cone resistance

q_t is the total cone resistance

s_{v0} is the total vertical stress

An N_{kt} assessment was performed to investigate the need to update the N_{kt} range for each geotechnical unit and provide a more accurate presentation of undrained shear strength from derived CPT data; the N_{kt} assessment is discussed further in Section 8.

4.5 Relative Density

When coarse grained cohesionless soils are encountered, relative density values are presented if appropriate. Relative density of the soil is a measure of the compactness of the soil and varies with the particle size and mineralogy; the gradation and the manner in which the soil mass was compacted (geological history). The relative density has been derived from the CPTU results using the correlation proposed by Jamiolkowski et al. 1988.

Relative density correlations are derived from laboratory tests carried on laboratory tests on normally consolidated (OCR=1), un-cemented, un-aged, clean, predominantly silica sands. If the tested sand stratum contains any deviation from this in the form of gravel, silt or clay content, slight cementation, or high carbonate content, then the estimated relative densities may be deemed to be un-representative. As this equation is derived from idealised conditions in a laboratory, the predicted relative density (Dr) results should be applied with caution and the results considered as “equivalent” values of relative density.

Relative density (D_r) is calculated using the following formula:

$$D_r = \left[\frac{1}{2.93}\right]\ln\left[\frac{q_c}{205\left(\sigma_m'^{0.51}\right)}\right]x100$$

where $$\sigma_m' = \left[\frac{\sigma'_{vo}\left(1+2K_0\right)}{3}\right]$$ Estimated mean effective stress at test depth

σ'_{v0} = Total effective overburden pressure of overlying sediments
K_0 = Coefficient of lateral earth pressure
qc = Measured Cone End Resistance

K_0 is the coefficient of lateral earth pressure; both K_0 values of 0.5 and 1.0 are used on CPTU logs to present a range of relative density values. Engineers should utilise engineering judgement when reviewing the relative density profiles. In general, the lower K_0 value represents normally consolidated conditions, usually encountered at shallow depths, and the higher K_0 value is more appropriate in overconsolidated soil conditions encountered at greater depth.

Table 4.4 presents the descriptive terms used in the soil descriptions where derived relative density is presented. All measured and derived CPTU data is presented on CPTU logs in APPENDIX 3.

Table 4.4 – Relative Density Classification

Relative Density Descriptor	D_r (%)
Very loose	0 – 15
Loose	15 – 35
Medium dense	35 – 65
Dense	65 – 85
Very Dense	85 – 100

5 Sampling Analysis

5.1 General

Sampling operations were conducted in 57 boreholes across Bornholm I and Bornholm II. Of these, 46 were downhole composite boreholes and 11 were sample-only.

Sampling comprised of piston sampling with a Shelby tube, Push sampling with Shelby tube, and coring. Coring operations were performed in over-consolidated sediments where the Shelby tubes could not penetrate the undisturbed soil and in all encountered rock layers to target depth. Both PQ3 and Geobore-S coring operations were carried out in the field.

Downhole sampling operations were carried out in accordance with the requirements of ISO 19901-8:2014. Additional details regarding sampling and coring operations can be found in the Field Operations Report.

5.2 Composite Borehole Results

In total 2939 samples and 955 cores were acquired from the field across Bornholm I and Bornholm II. 189 samples were retained in their Shelby Tubes for onshore extrusion. The sample retained in tube BH-216 P16T1 was cancelled as the material was drill cuttings and not representative.

All data acquired in the field was entered into Gardline's proprietary software TerraFusion to create the preliminary composite borehole logs showing the unitisation of the soil encountered along with all the offshore laboratory test results. Preliminary logs were then updated using the results of the onshore laboratory testing programme.

Combined locations were also created to show all the information from the composite borehole and seabed CPTU location together. Layer boundaries presented in the combined locations were adjusted where required to align with the seabed CPTU data profile. All logs can be found in APPENDIX 2.

5.3 Sampling Challenges

Due to client representative concerns, the core samples acquired from BH-207 were not split with an exact saw in the field while a safety concern was investigated. As a result, the drilling fluid in the liners saturated the cores which made them unsuitable for testing when they arrived at Gardline's onshore laboratory.

6 PS Logging

6.1 General

PS Logging data was acquired at six locations; no retests were required. The M.V. Kommandor Susan conducted PS logging activities on BH-206, BH-202 and BH-103. The M.V. Ocean Vantage conducted PS logging activities on BH-104_a, BH-108 and BH-216_a. Equipment was managed by Robertson Geologging Ltd operators and Gardline Ltd operators. Data quality during PS logging acquisition was overseen by Robertson Geologging Ltd.

6.2 Equipment

PS Logging is used for the in-situ determination of compression (P) and shear (S) wave seismic velocities. The equipment, manufactured by OYO Corporation, comprises a directional seismic source and a pair of directional seismic detectors mounted together with associated power, switching and data transmission electronics, in a 6.25m long wireline sonde (Figures 6.1 and 6.2). It is deployed in a fluid filled uncased borehole from a logging winch (Figure 6.3) fitted with a depth encoder. Operation is controlled using Robertson Geologging Ltd (RGL) software running a RGL Micrologger 2 logging interface unit.

Figure 6.1 and Figure 6.2 – PS Logging Sonde Components

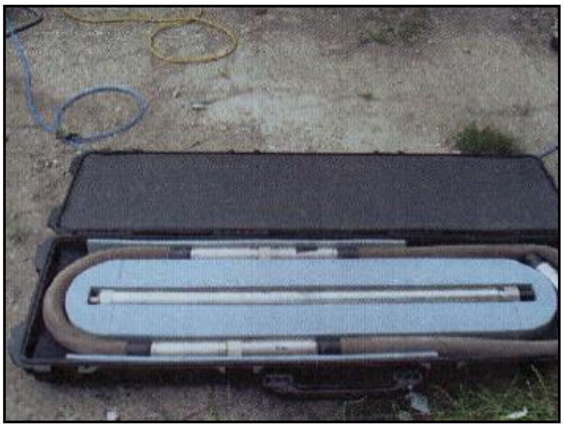


Figure 6.3 – Robertson Geologging Ltd Winch

6.3 Summary of PS Logging Operations

In operation, the seismic source in the sonde is activated to produce a sequence of seismic pulses which excite 'flexural' waves. Depending on the direction of impulse, the seismic waves that are generated travel at the P and S-wave velocities of the formation and are detected by the seismic receivers which are 1.00m apart. The difference in arrival time at the lower and upper receivers can be measured from the displayed waveforms and the seismic velocities can then be calculated.

It is necessary for the operator to control the system settings to ensure that the data recorded is of sufficient quality for the arrival time measurements to be carried out. Measurements were made from the base of the borehole, upwards at 1m intervals to mudline. Once specified depths have been measured, the data is then processed. A Raw data file is stored for each record. From the resulting wave formation plots, the first arrival times from both the compression (P) and shear (S) wave velocities are picked. The seismic velocities are then determined and used along with other geotechnical data such as densities to aide in characterisation of the basic material properties and determination of G_0.

PS Logging interpretation is a manual process completed by experienced engineers. They are able to use amplitude gains and digital filters during the processing to assist them picking the correct location on the waveform. After this data is picked it is sent back to RGL where a Senior Geophysical Engineer will check the picks made. Picks will be adjusted as necessary or approved as good. A confidence table is also filled in to show what confidence the engineer has in their pick.

6.4 PS Logging Data Collection Operations

PS logging acquisition was scheduled from base of borehole to mudline at six locations; due to the ground conditions of the borehole and associated risk of borehole collapse, this was not obtainable, and a more pragmatic approach was undertaken.

A preliminary borehole log for each location was utilised before testing commenced to plan the data acquisition strategy. It was decided, due to the potential of borehole collapse, logging would be carried out in stages. The API drill string was removed in sections at a time to reduce the chance of borehole collapse during testing; each stage is referred as a Run. A breakdown of the length of each run can be found in the PS logging data quality tables in APPENDIX 5. No rathole was drilled.

Table 6.1 provides a summary of the depths PS logging data was collected.

Table 6.1 – PS Logging Location Summary

Borehole ID	Start Depth (m)	End Depth (m)	Number of Runs
BH-103	64.00	4.00	3
BH-104_a	55.00	2.00	4
BH-108	53.00	5.00	4
BH-202	61.00	5.00	3
BH-206	56.00	5.00	3
BH-216_a	46.00	6.00	4

Both P and S wave velocities were picked and represented in the log. Processing of the PS Logging data was completed by Robertson Geologging.

6.5 PS Logging Field Summary - QA

Data acquired from the PS Logging operations in the field was quality checked by Robertson's Geologging who have extensive experience in PS logging with dedicated engineers for the operations. No issues with data acquisition and processing were observed in the field. All data acquired and processed by Robertson's Geologging was later reviewed by Gardline's onshore project engineers to ensure high data quality and reliability.

6.6 PS Logging Results

Robertson Geologging provided tables of PS logging data quality for each depth where P and S wave data signals were obtained. Tests at each depth were assigned a trace class ranging from A to E. Table 6.2 summarises the different classes of data quality and their meaning.

Table 6.2 – PS Logging Trace Class Summary

Trace Class	Description
A	Excellent, strongly defined arrivals, high coherence
B	Good, arrivals defined, somewhat incoherent
C	Poor, arrivals are pickable, but somewhat undefined, minimal coherence
D	Very poor, best estimate
E	Not used

All results provided by Robertson Geologging, including the tables of data quality, can be found in APPENDIX 5.

7 Geodetic Information and Water Depths

7.1 Equipment

Primary navigation for this project was the C&C Technologies C-Nav DGNSS system using their network of reference stations. Dual independent receivers were used in order to give full redundancy on the systems. The vessel operates with three DGNSS receivers as summarised in Table 7.1 below:

Table 7.1 - GNSS System Overview – Kommandor Susan and Ocean Vantage

DGNSS Receivers		
1	C-Nav Stbd	Oceaneering CNav corrections
2	C-Nav Port	Oceaneering CNav corrections
3	Hemisphere VS110	SBAS EGNOS Europe corrections

7.2 Geodetic Information

A summary of the geodetic information can be found in the Survey Positioning Reports presented in the Field Operations Report.

7.3 Water Depth Measurements and Locations

The water depths were obtained by SBES (Single Beam Echo Sounder). Depths are to the vertical datum DTU21MSL (Mean Sea Level) in metres and is presented on logs in APPENDIX 2.

8 Field Unitisation and Geological Cross Sections

8.1 General

Geotechnical unitisation was first performed by dividing the soils by depositional environment, determined through material and mechanical properties, and supported by geophysical survey information. Once the soils and rocks had been grouped by depositional environment, they were further sub-divided by assessing the material of the soil. Both site areas, Bornholm I and Bornholm II, contain the same geotechnical units as summarised in Table 8.1 below:

Table 8.1 – Geotechnical Units Summary

Formation	Geotechnical Unit	Description
1	1a	Clay, Gyttja, Organic Clays
	1b	Sands, Silts and Silty Sands
2	2a	Silty Clays
	2b	Sands and Silts
3	3a	Clay Tills and Clays
	3b	Sand Tills, Silt Tills and Massive Sands
4	4a	Limestones and Sandstones
	4b	Mudstones, Marlstones and Siltstones
	4c	Chalks

Environment of deposition was determined through analysis of the material properties of the soil. The environments interpreted for each unit was supported using information from the geophysical survey and the geological history of the Baltic Sea. The derivation of geological age was supported using the information from the geophysical survey and the results of the palynology and micropalaeontology testing. The age and environment of deposition for each formation are summarised in Table 8.2.

Table 8.2 – Depositional Environment Summary

Formation	Environment of Deposition	Age
1	Marine (Ma)	Postglacial (Pg) – Present day to 11500 years B. P.
2	Glacier / Meltwater (Gl/Mw)	Postglacial / Late Glacial (Pg/Lg) – 11500 to 15000 years B. P.
3	Glacier (Gl)	Glacial (Gc) – 15000 to >22000 years B. P.
4	Marine (Ma)	Cretaceous / Jurassic (Ct/Jr)

Tables showing ranges of geotechnical parameters for each geotechnical unit are presented in APPENDIX 1. Tables present ranges of classification, chemical and rock strength test results for Bornholm I and Bornholm II separately, and includes the number of tests conducted within each unit (N) and the average results (Av). The data ranges correlate well with the assigned units, providing additional evidence that the interpretation of the geotechnical units is realistic.

8.2 Biostratigraphic Analysis

An assessment of the palynology and micropaleontology was undertaken on three selected samples across the Bornholm I and Bornholm II site areas. The presence and abundance of terrestrially derived spores, pollen, and freshwater algae, and marine dinocysts were identified, and corresponded to the period of time they inhabited. A summary of findings is summarised in **Table 8.3** below.

Table 8.3 – Biostratigraphic Analysis Summary

Sample ID	Description	Palynology	Micropaleontology	Comments
BH-105 P10Q1	PEAT, sandy, black (2.5Y 4/2), slightly calcareous, micaceous with extremely closely to closely spaced thin to thick laminations of fibrous black organics and staining. (Dense to very dense)	A high abundance assemblage of palynomorphs, dominated by a variety of trilete spores.	Marine foraminifera, Radiolaria and inoceramid bivalve taxa identified.	Interpreted as glacially disturbed sediment based on varying age ranges of observed taxa however confidence is reduced by absence of younger (Tertiary) taxa, namely angiosperm pollen.
BH-115 P57B1	COAL, low to medium grade, black (2.5Y 2.5/1)	A high abundance assemblage of palynomorphs, dominated by a variety of trilete spores.	N/A	Palynological evidence suggests an age distribution of Jurassic/Early Cretaceous age, however an alternative interpretation is that of Mesozoic (coal) material reworked into disturbed Quaternary glacial sediment.
BH-214 P89B1	PEAT, decomposed, clayey, black (2.5Y 2.5/1), non-calcareous	A high abundance and high diversity assemblage dominated by a variety of angiosperm pollen, with additional fern spores and freshwater algae.	No microfauna present.	Palynological evidence suggests this sample represents Quaternary glacially disturbed sediments. The paleoenvironment determined from the palynology is consistent with an interglacial setting. This interpretation may also represent remobilised sediments as a result of glacial activity.

8.2.1 Summary

Soils identified by biostratigraphic analysis suggest Jurassic ages, however preliminary interpretation includes uncertainty introduced by geological history of glacial reworking and tectonic setting. Block faulting associated with strike-slip faulting complicates interpretation of a single sample. Further analyses are likely necessary to develop a holistic understanding of the site area. Unit 4 has been interpreted as containing deposits of Jurassic/Early Cretaceous via supporting palynological taxa evidence. Due to the limited distribution of Palynology and Micropaleontology testing conducted across Bornholm I and Bornholm II, further biostratigraphic analyses of in situ bedrock is likely necessary to confidently identify Jurassic and Cretaceous rock, the extent of which was identified only in the easternmost of seismic lines undertaken across Bornholm I and Bornholm II during the geologic desk study report.

Whilst BH-115 P57B1 was found to be absent of taxa definitively younger than Cretaceous in age (namely angiosperm taxa generally ubiquitous with the Tertiary/Quaternary), the analysed sample was composed entirely of the (Mesozoic) coal deposit. To determine if the identified taxa are representative of the in situ palaeoenviroment (and further age) it is likely necessary therefore, as the analyses suggests, to conduct further testing deeper and/or shallower than BH-115 P57B1 within the stratigraphic column. Further confidence could be attained through testing within rock core samples of Unit 4 identified in **Table 8.1**.

8.3 Bornholm I and Bornholm II Geotechnical Unit Descriptions

8.3.1 Bornholm I

Unit 1 most commonly consists of organic-rich clays and gyttja, representative of a recent lacustrine or marine environment. Less commonly, sands and silty sands may be the dominant sediment type in this unit. Unit 1 represents the recent deposits after the glacial period of the Baltic Sea. Unit 1 was found in thicknesses of 0.1-3m in the southwest of the site, generally increasing in extent in the central site area up to a thickness of 9m. This trend of Unit 1 deepening continues northeasterly, with the maximum unit thickness encountered at BH-115 (17.31m).

Unit 2 most commonly consists of silt-rich clays with occasional dense fluvial sand deposits. Unit 2 represents the Postglacial lacustrine deposits underlying the Postglacial deposits of Unit 1. Unit 2 was absent from the south-westernmost borehole locations. Further northeast Unit 2 was found to deepen in a southwest to northeast trend up to approximately 11m, apart from BH-106 where the unit was more limited in extent. In the central site area the extent of Unit 2 was found to be variable, most commonly 1-5m in thickness but occasionally >10m. In the northeast of the site area Unit 2 was commonly found between 1-2m in thickness. Unit 2 was found to be most extensive in north-easternmost location BG-115 at approximately 15m in thickness.

Unit 3 most commonly consists of clay tills, gravelly clays and massive formations of sand. Unit 3 represents Late/Postglacial deposits, with the massive sand deposits interpreted as glaciofluvial in origin. The clays of Unit 3 are distinguished from those in Unit 2 by characteristically higher shear strengths and an increase in gravel content. Unit 3 was encountered at depths shallower than 10m below mudline in the southwest of the site, including from mudline at southernmost BH-101. Extent was found to be highly variable, with thickness of the unit ranging from approximately 5m-45m. In central borehole locations the unit deepened and became more extensive along a northeasterly trend, except for locations BH-108 and BH-109 where thickness was approximately 3m and Mesozoic bedrock was encountered more shallowly. In the northeast of the site area the extent of Unit 3 continued to show lateral variability, with thickness of unit commonly around 10m. At the easternmost borehole locations where Unit 4 was not encountered thickness increased to approximately 40-50m.

Unit 4 represents the bedrock, consisting of limestones, mudstones, sandstones and chalk. This unit represents the marine deposits of the Cretaceous/Jurassic, supported by the palynological evidence obtained from analysed Mesozoic coal recovered from BH-115. Bedrock was encountered variably in the southeast of the site, most commonly around 10m below mudline but on occasion deeper than 30m below mudline. The surface of Unit 4 was equally found to be variable in the central site area, most often between 7-15m below mudline. In the northeast of the site the surface of the bedrock deepened to approximately 20m below mudline. In the north-easternmost boreholes Unit 4 was not encountered within the 70m borehole extent.

8.3.2 Bornholm II

Geotechnical units encountered in the Bornholm II site are similar to that of Bornholm I, with the exception of the presence of erratics, where large amounts of bedrock appear to be suspended within the soil units of Unit 3. An example of this can be seen in BH-205 at 19m. A possible interpretation of this observation is a more extensive seismic history of fault block movement via strike-slip faulting in Bornholm I than Bornholm II. Another potential explanation is the bedrock could have been rafted or transported by (fluvio) glacial activity.

Unit 1 was found to be highly variable in extent over the site area. Unit 1 was generally found between 6-9m thickness in westernmost boreholes, with intermittent shallowing of approximately 2-4m in extent, to a maximum thickness of around 12.5m In the central site area Unit 1 was generally 5-8.5m in extent, except for at BH-211 where extent was around 2m. Boreholes on the northern site boundary (BH-213 and BH-214) showed shallowing of Unit 1 to 1-3m in the extent. In the northeast of the site Postglacial deposits were variable in thickness, generally <10m in extent.

Unit 2 was found to be variable in extent in the southwest of the site area, typically greater than 3m and less than 10m in thickness. In the central site area the unit appeared to thin in extent, with thickness of around 1-2m. In the northeast of the site area extent continued to be limited, apart from north-easternmost BH-220 where the maximum extent was observed (35m+).

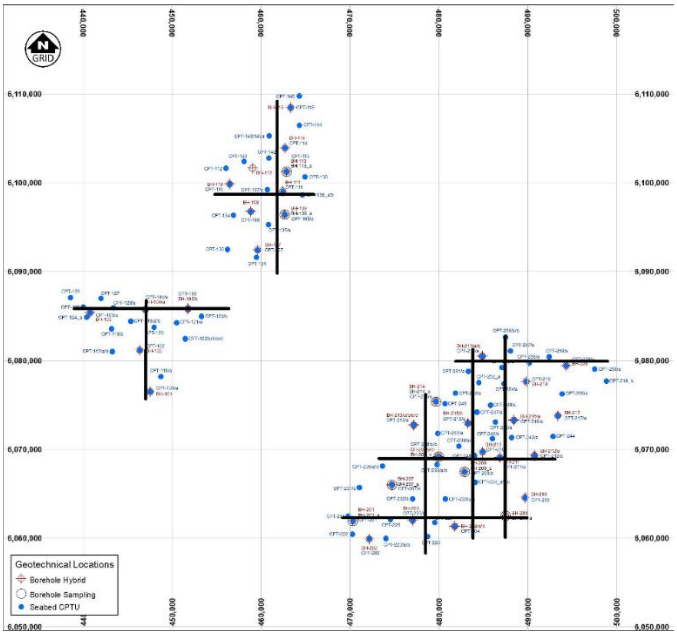
Unit 3 was found to be highly variable in extent over the site area. In the southwest of the site Unit 3 was found with thicknesses >55m where bedrock was not encountered; elsewhere between 1-6m where the surface of the bedrock was encountered at shallow depths. This lateral variability persists in the central, northern and eastern site areas. Unit 3 was found notably thin at BH-219 (approximately 4m in extent) where other northern boreholes encountered 20m+.

Unit 4 was found to be highly variable in extent over the site area. In the southwest of the site bedrock was encountered between 10m and 66m below mudline, most often 10-20m below mudline. This variability persisted into the central, northern, and eastern borehole locations, with the unit absent within the 70m borehole depth at several locations. A possible interpretation of this is this deepening is the result of erosional features/scour associated with glaciofluvial erosion.

8.4 Geological Cross Sections

Geological cross-sectional profiles were produced to illustrate the lateral variability of the units encountered across Bornholm I and Bornholm II. Two N-S / W-E cross sections, and three N-S / W-E cross sections were taken at Bornholm I and Bornholm II respectively. The location of these cross sections were selected to present as many locations as possible, within 1km of the alignments. The locations of the geological cross sections are shown in Figure 8.1 below:

Figure 8.1 – Geological Cross Sections Locations



The geological cross sections illustrate the lateral variations in unit thickness across the Bornholm I and Bornholm II fields. The cross sections were produced to also present the CPT cone end resistance as this supported the interpretation of the unit boundaries.

The geological cross sections are presented in APPENDIX 2.

8.5 N_{kt} Assessment

An N_{kt} assessment was performed on each appropriate geotechnical unit by correlating the in situ CPTU data profiles with undrained shear strength obtained from onshore triaxial laboratory test results. N_{kt} assessments were conducted in units 1a, 2a and 3a.

Another assessment was performed for Direct Simple Shear and Undrained Unconsolidated triaxial tests to assess shear strength anisotropy of the soil. Performing the assessment on Unconsolidated Undrained triaxial tests allows for the visualisation of the discrepancy between undrained shear strengths obtained from consolidated and unconsolidated testing. The variability in undrained shear strength of the soil with regards to the type of loading is called anisotropy. The anisotropy factors, defined respectively as s_{uDSS}/ s_{uC}, s_{uE}/ s_{uC}, and $s_{u,UU}$/ s_{uC}, where s_{uDSS}, s_{uC}, and $s_{uE,}$ are the undrained shear strengths obtained from DSS, Consolidated Compression, and Consolidated Extension triaxial tests which can be calculated directly from the ratios between N_{kt} values.

Table 8.4 presents the statistical result summary of the Nkt assessment for the total site area, calculated for different type of test, as well as the calculated anisotropy factors.

Table 8.4 – N_{kt} Assessment Results

Bornholm I											
Unit 1a				Unit 2a				Unit 3a			
Test	Number	Average N_{kt}	Anisotropy	Test	Number	Average N_{kt}	Anisotropy	Test	Number	Average N_{kt}	Anisotropy
CAUC	1	18	-	CAUC	2	34	-	CAUC	1	19	-
CIUC	2	18	-	CIUC	4	18	-	CIUC	N/A	N/A	-
Compression	3	18	1.00	Compression	6	23	1.00	Compression	1	19	1.00
DSS	1	28	0.64	DSS	1	42	0.55	DSS	1	54	0.35
UU	19	26	0.68	UU	9	23	0.99	UU	3	23	0.81
Bornholm II											
Unit 1a				Unit 2a				Unit 3a			
Test	Number	Average N_{kt}	Anisotropy	Test	Number	Average N_{kt}	Anisotropy	Test	Number	Average N_{kt}	Anisotropy
CAUC	2	25	-	CAUC	3	15	-	CAUC	1	7	-
CIUC	N/A	N/A	-	CIUC	2	38	-	CIUC	2	29	-
Compression	2	25	1.00	Compression	5	24	1.00	Compression	3	22	1.00
DSS	N/A	N/A	N/A	DSS	1	20	1.21	DSS	N/A	N/A	N/A
UU	15	23	1.09	UU	9	27	0.88	UU	3	37	0.58

Due to the nature of the downhole composite boreholes, CPT data was not acquired at the same depth at which undisturbed samples were taken. Therefore, a direct comparison could not be made between the onshore laboratory test results and downhole CPT data. Attempts were made to extrapolate the downhole CPT data around the depths where undisturbed samples were acquired but this proved to yield unrepresentative N_{kt} values, so this approach was abandoned. To resolve this issue, data from the corresponding seabed CPTU locations was inferred on the undisturbed samples at shallower depths to calculate reliable N_{kt} values to be used for the assessment.

The results of the assessment showed low correlation due to the variability of undrained shear strength. Unit 3A's calculated N_{kt} values are considered less reliable, as the majority of seabed CPTU's would end before or at the top of Unit 3A, resulting in there not being CPT data on the corresponding with the depths at which undisturbed samples were obtained. As a result, the N_{kt} assessment is considered inconclusive, and the values have not been changed from the original 12.5-16.5 and 15.0-20.0 N_{kt} ranges used when producing the preliminary logs. These values are based on N_{kt} ranges that were acquired on similar soils within the Baltic Sea, obtained on previous projects.

Additional downhole CPT data is required to improve this assessment.

Visual representations of the N_{kt} assessments can be found in APPENDIX 8.

8.6 Equipment Recommendations for Future Works

During project fieldwork, operational issues were encountered due to the soft seabed sediments across the Bornholm I and Bornholm II fields. Specific details regarding these issues are discussed in the Field Operations report. This should be considered for future site investigations, as the soft seabed sediments provide limitations on what equipment can be deployed for data acquisition.

Due to the abundance of rock material encountered across the Bornholm I and Bornholm II site areas, it is recommended that the application of Geobore-S coring techniques should be utilised to acquire high quality core samples for representative logging and testing purposes.

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